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DEVELOPMENT OF A RATIONAL DESIGN PROCEDURE FOR OVERLAND FLOW SY—ETC(U)  
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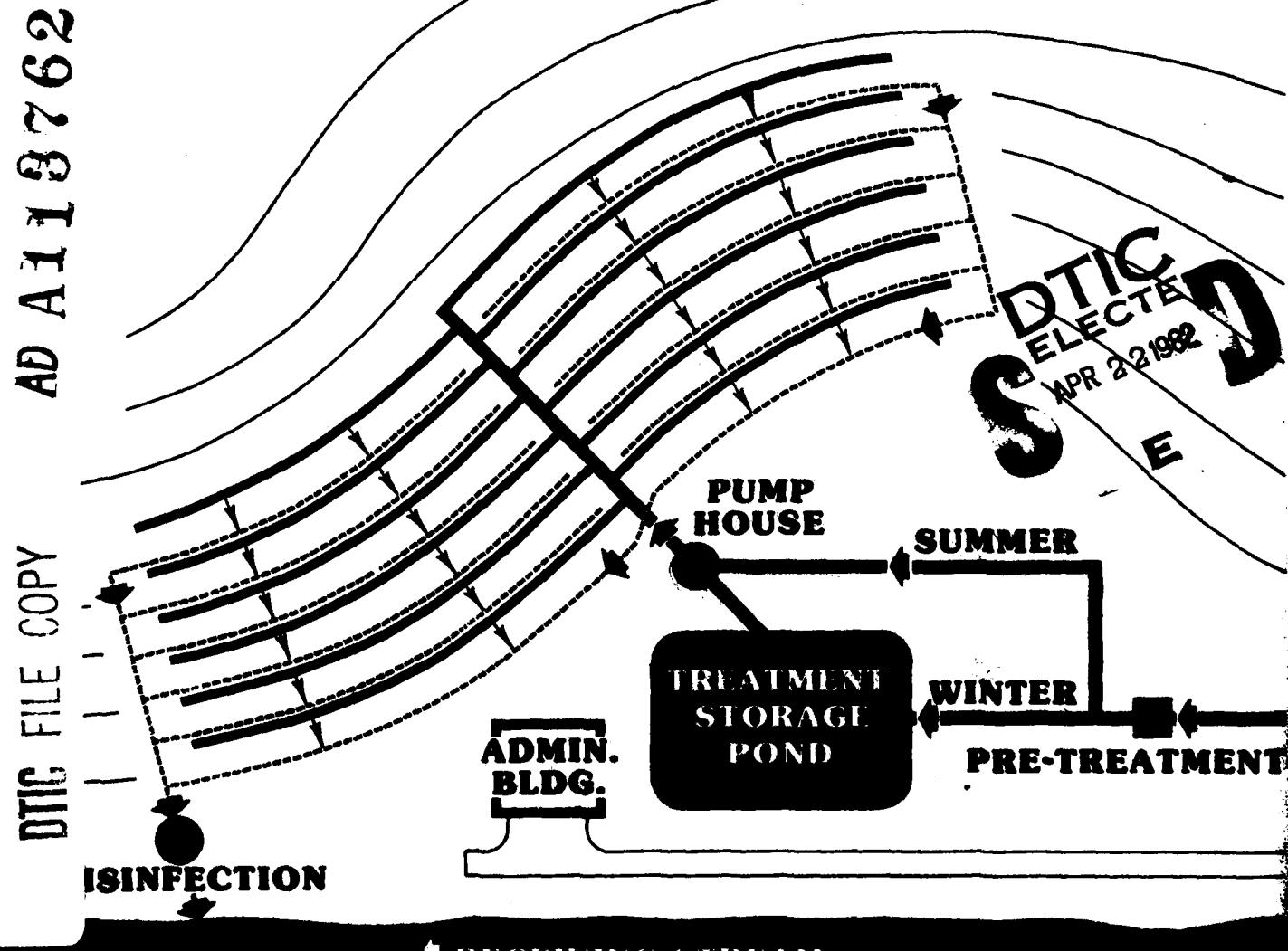


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*Development of a rational design procedure  
for overland flow systems*



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## OVERLAND FLOW RESEARCH REPORTS

This is one of a series of reports on wastewater treatment by overland flow published by the U.S. Army Corps of Engineers under the Land Treatment of Wastewater Research Program. Other published and available reports on this topic are listed below.

Carlson, C.A., P.G. Hunt, T.B. Delaney (1974) Overland flow treatment of wastewater. *Misc. Paper Y-74-3*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Chen, R.L. and W.H. Patrick, Jr. (1980) Nitrogen transformations in a simulated overland flow wastewater treatment system. *CRREL Special Report 80-16*.

Hoeppel, R.E., P.G. Hunt, T.B. Delaney (1974) Wastewater treatment on soils of low permeability. *Misc. Paper Y-74-2*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Jenkins, T.F. et al. (1979) Prototype overland flow test data: June 1977-May 1978. *CRREL Special Report 79-35*.

Jenkins, T.F., D.C. Leggett, C.J. Martel and H.E. Hare (1981) Overland flow: Removal of toxic volatile organics. *CRREL Special Report 81-1*.

Lee, C.R. et al. (1976) Highlights of research on overland flow for advanced treatment of wastewater. *Misc. Paper Y-76-6*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Martel, C.J., T.F. Jenkins and A.J. Palazzo (1980) Wastewater treatment in cold regions by overland flow. *CRREL Report 80-7*.

Peters, R.E., C.R. Lee and D.L. Bates (1981) Field investigations of overland flow treatment of municipal lagoon effluent. *Technical Report EL-81-9*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

*Cover: Possible configuration of terraces and storage pond for hypothetical overland flow system.*

# CRREL Report 82-2

February 1982

## *Development of a rational design procedure for overland flow systems*

C.J. Martel, T.F. Jenkins, C.J. Diener and P.L. Butler

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report describes the development of a new design procedure for overland flow systems that is based on hydraulic detention time, a familiar concept in wastewater treatment process design. A two-year study was conducted at Hanover, New Hampshire, on a full-scale overland flow site to obtain performance data in relation to detention time. Kinetic relationships were developed for removal of biochemical oxygen demand, total suspended solids, ammonia, and total phosphorus. Also, an empirical relationship was developed to predict hydraulic detention time as a function of application rate, terrace length, and slope. These relationships were validated using published data from other systems. An example of how to use the new procedure and a comparison with the conventional design approach is included. A major advantage of the new procedure, which should significantly reduce site preparation costs, is that it allows overland flow systems to be designed for a wide range of site conditions as long as detention time requirements are met.		

## PREFACE

This report was prepared by C.J. Martel, Environmental Engineer, of the Civil Engineering Research Branch, Experimental Engineering Division, T.F. Jenkins, Research Chemist, of the Earth Sciences Branch, Research Division, and by C.J. Diener, Civil Engineering Technician, and P.J. Butler, Physical Science Technician, of the Civil Engineering Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding for this research was provided by Corps of Engineers Civil Works Project CWIS 31732, *Land Treatment Management and Operation*.

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# DEVELOPMENT OF A RATIONAL DESIGN PROCEDURE FOR OVERLAND FLOW SYSTEMS

C.J. Martel, T.F. Jenkins, C.J. Diener and P.L. Butler

## INTRODUCTION

### Background

An overland flow system consists of a series of grassy terraces which are carefully graded so that wastewater flows downslope in a thin sheet. Gated pipe, slotted troughs, or sprinklers are used to uniformly distribute the wastewater at the top of each terrace, and a ditch or channel collects the runoff at the base of the terrace (Fig. 1). When properly designed and managed, the quality of runoff from an overland flow system can easily meet secondary effluent standards.

Unlike most wastewater treatment systems, overland flow reduces the volume of water to be discharged. This reduction in volume is caused by losses through evapotranspiration and percolation as wastewater flows over the soil surface. Typically, the volume of runoff is 60 to 90% of that applied. Consequently, removal efficiency should be calculated on a mass rather than concentration basis.

Overland flow systems offer a number of advantages over conventional treatment. First and foremost is the lower cost of operation and maintenance. Highly skilled personnel are not needed to run the facility and energy requirements are significantly lower (Middlebrooks and Middlebrooks 1979). Overland flow systems are also very reliable and able to withstand large variations in strength of applied wastewater without system upset (Aly et al. 1979). Another important advantage is that no sludge is produced except by pretreatment processes. Overland flow systems can also provide

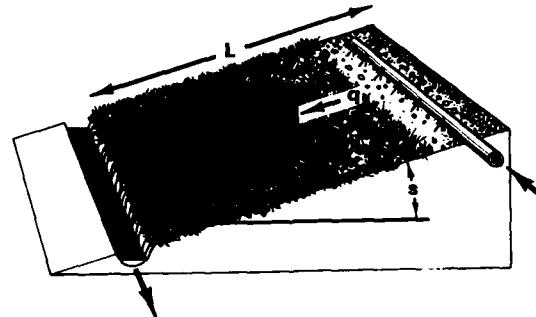


Figure 1. Concept drawing of overland flow system (where  $L$  is terrace length,  $q$  average overland flow rate and  $s$  the slope).

an economical return in the form of high quality forage crops. Palazzo et al. (in prep.) estimated the value of this crop to be  $\$858 \text{ ha}^{-1}$  per year.

In spite of these advantages, not many overland flow systems have been built. One of the main reasons is the lack of a rational procedure for design. The procedure presented in U.S. EPA (1977) is based on general guidelines which are difficult for inexperienced engineers to interpret. For example, when untreated or primary effluent is applied, the designer is advised to select a hydraulic loading rate within a range of 6.4 to 20  $\text{cm wk}^{-1}$ , depending on climate, degree of treatment and detention time. Without previous experience it is difficult for the designer to

select a valid loading rate within such a wide range. Also, little information is given on how other variables such as terrace length and slope can affect performance. Thus, a more comprehensive and rational design procedure, which takes these factors into account, is needed to assure that discharge requirements are met.

The new design procedure developed in this report is based on reactor kinetics, a concept familiar to most environmental engineers. In the case of overland flow, the reactor is the soil surface where various physical, biological and chemical reactions take place. As in conventional process design, the controlling parameter is detention time. For overland flow, detention time is the average time a unit volume of water takes to travel from the top to the bottom of the terrace. The desired level of treatment can be achieved by controlling the length of time that wastewater remains in contact with the soil surface. With this approach, overland flow systems can be constructed for a wide range of site conditions as long as detention time requirements are met. This would significantly reduce site preparation costs.

#### Objectives

A design procedure based on detention time requires knowledge of two basic relationships. First is the hydraulic relationship among application rate, site characteristics and detention time. With this relationship the designer can determine the application rate needed to satisfy detention time requirements. Second is the kinetic relationship

between detention time and removal of biochemical oxygen demand (BOD), total suspended solids (TSS), ammonia ( $\text{NH}_3\text{-N}$ ) and total phosphorus (total P). With these relationships, the designer can determine the detention times needed to achieve the desired removal efficiency.

The specific objectives of this study were to

1. Develop a method which can be used to predict the hydraulic detention time.
2. Determine the removal kinetics for BOD, TSS,  $\text{NH}_3\text{-N}$  and total P.
3. Validate the detention time and kinetic relationships using data from other systems.
4. Provide an example using the new design procedure.

#### Scope

Data used in the development of the design procedure were obtained from the CRREL overland flow test site in Hanover, New Hampshire, during the 1978 and 1979 growing seasons (April through October). All the kinetic relationships were developed using primary effluent; use of this procedure for design of overland flow systems receiving a secondary or lagoon effluent will be discussed later in the report.

The hydraulic detention time relationship developed at CRREL was validated using data from the Utica, Mississippi, overland flow site (Peters et al. 1981) and the pilot scale system at the University of California, Davis (Smith et al. 1980). Data from several other domestic and foreign overland flow systems were used to validate the kinetic relationships.

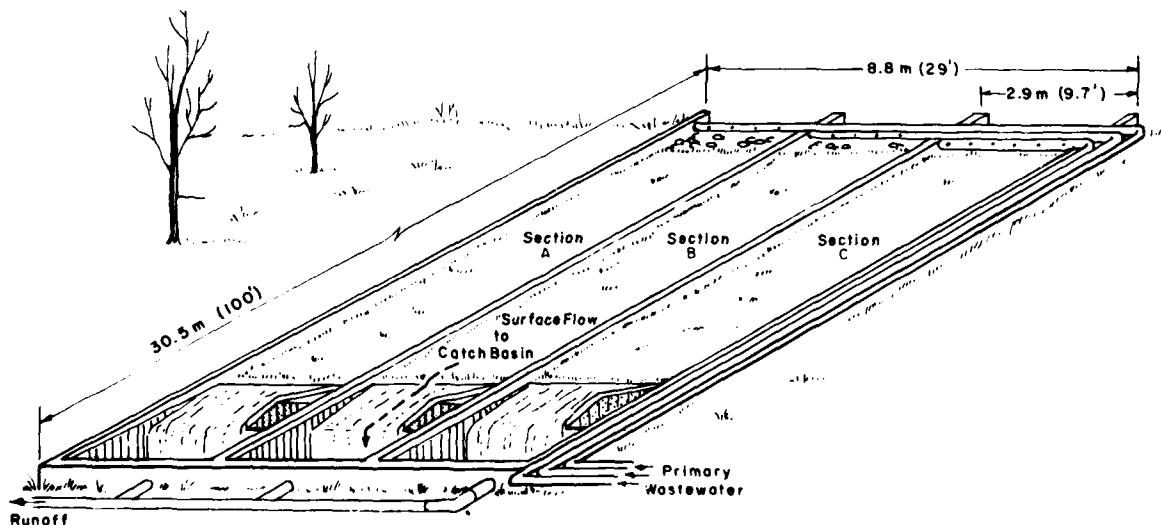


Figure 2. Schematic of CRREL overland flow test site.

## DESCRIPTION AND OPERATION OF CRREL OVERLAND FLOW TEST SITE

The CRREL overland flow test site has been in operation since June 1977. This site is 30.5 m long x 8.8 m wide (0.03 ha) and graded to a 5% slope. It is subdivided into three equal sections designated A, B and C so that parallel studies can be conducted. A schematic of the site is shown in Figure 2.

Soil on the site is classified as a Hartland silt loam with sand, silt and clay contents of 5, 72 and 23% respectively. The cation exchange capacity is 5 meq/100 g and the pH is 7.1. Underlying the soil at a depth of 15 cm is a 30.0-mil-thick rubber membrane, which was installed to prevent downward percolation. The grass cover on the site is a mixture of many species including K-31 tall fescue, orchard-grass, Kentucky bluegrass and quackgrass (Palazzo et al. 1980). The grass was harvested on the average of once every six weeks during the growing season.

Undisinfected primary effluent was applied to the overland flow test site during the entire study. Perforated plastic pipe was used to distribute wastewater along the top of each section, and a bed of crushed stone placed beneath the pipe helped to uniformly disperse the flow. The quality of the primary effluent is shown in Table 1.

The application rate to each section was monitored and controlled by means of a constant head weirbox. Five application rates ranging from 0.35 to 1.20 m<sup>3</sup> hr<sup>-1</sup> were tested. The application cycle was 7 hr on, 17 hr off for 5 days per week. At these application rates and this cycle, the equivalent hydraulic

loading rates were 13.8 to 46.7 cm wk<sup>-1</sup>. Each application rate was evaluated for a period of approximately 6 weeks. All sections were operated simultaneously at the same application rate. Because of leaks in the membrane along the outside boundary, wastewater applications to section A were discontinued during 1979.

Runoff was collected at the base of each section in individual galvanized steel catch basins. A small submersible sump pump located in each basin discharged the runoff into a drainage ditch. The volume of runoff was recorded by flowmeters attached to the discharge lines. During this study, the average runoff rate was 75, 87 and 89% of the application rate for sections A, B and C respectively.

All measurements of detention time and water quality sampling were conducted during periods of hydraulic steady-state operation. The hydraulic steady-state period began when the runoff rate stabilized and it terminated when application was stopped. A typical runoff hydrograph is shown in Figure 3. In this case the hydraulic steady state period began 1 hr after commencing application and terminated 6 hr later when the system was shut down. The amount of time needed to reach hydraulic steady state varied depending on antecedent moisture conditions.

Hydraulic detention time was determined by measuring the travel time of a chloride tracer. Chloride was selected as a tracer because it is conservative and easily analyzed. A tracer solution was made by dissolving 94.6 g of sodium chloride in 3 L of distilled water. The sodium chloride solution was added as a "slug addition" to the distribution chamber

Table 1. Quality of applied primary effluent.

Parameter	Mean	Standard deviation	No. of observations
BOD (mg L <sup>-1</sup> )	72	23	58
Total suspended solids (mg L <sup>-1</sup> )	59	30	98
Total Kjeldahl nitrogen (TKN) (mg L <sup>-1</sup> )	36	10	32
Ammonia (mg L <sup>-1</sup> as N)	24	6	99
Nitrate (mg L <sup>-1</sup> as N)	0.05	0.15	98
Total phosphorus (mg L <sup>-1</sup> as P)	6.6	2.2	33
Orthophosphate (mg L <sup>-1</sup> as P)	4.8	1.4	31
Turbidity (JTU*)	31	13	100
pH	7.2	0.1	100
Specific conductivity (μmhos/cm)	513	105	40
Chloride (mg L <sup>-1</sup> )	28	6	30
Fecal coliforms (no./100 mL)	2.5 × 10 <sup>6</sup>	1.7 × 10 <sup>6</sup>	14
Fecal streptococci (no./100 mL)	0.2 × 10 <sup>6</sup>	0.2 × 10 <sup>6</sup>	7

\*Jackson turbidity units

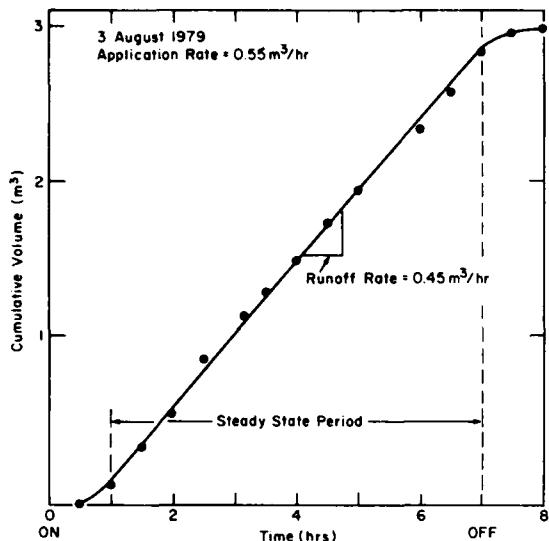


Figure 3. Typical runoff hydrograph.

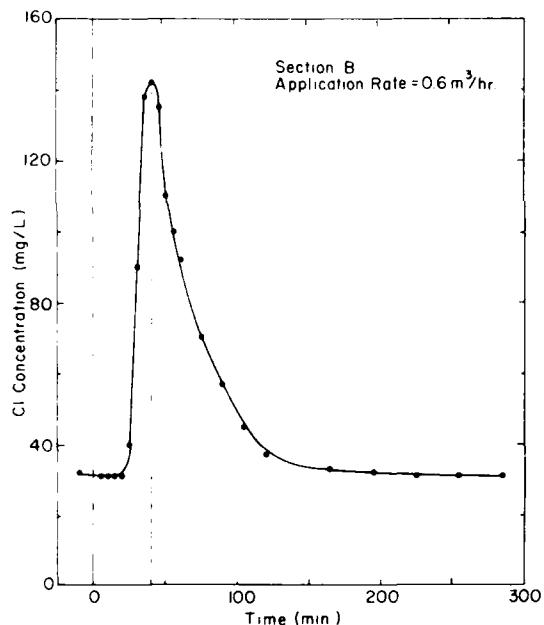


Figure 4. Typical chloride response curve for measuring detention time.

in the constant head weirbox. Composite samples were taken of the runoff at various intervals and analyzed for chloride. Chloride concentrations were then plotted vs time, and the peak of the response curve was chosen to represent detention time. The center of mass was not chosen to represent detention time because 36% of the tracer was lost in the percolate and plants. An example of a typical chloride response curve is shown in Figure 4. The detention time in this case was 40 min for an application rate of 0.6

Table 2. Parameters measured and frequency of analysis.

Parameter	Analyses/wk *
BOD	3
TSS	5
TKN	1
NH <sub>3</sub> -N	5
NO <sub>3</sub> -N	5
Total P	1
PO <sub>4</sub>	1
Turbidity	5
pH	5
Specific conductivity	1
Chloride	1
Fecal coliforms	1
Fecal streptococci	1

\*Based on five application periods per week.

$\text{m}^3 \text{ hr}^{-1}$ . Altogether, 50 detention times were measured at the CRREL site during this study (see App. A).

Flow-proportioned composite samples were taken of the applied primary effluent and runoff during each application period. Primary effluent samples were taken at fixed time intervals with an automatic composite sampler. Runoff samples were taken by small peristaltic pumps which were activated by relay switches during each operating cycle of the catch basin's sump pump. The water quality parameters measured and the frequency of analysis are shown in Table 2. Analytical techniques are discussed in Appendix B.

## HYDRAULIC DETENTION TIME

The hydraulic detention time on an overland flow terrace is dependent on many factors including application rate, slope, length of terrace, surface microtopography, soil infiltration rate, evapotranspiration and vegetation density. Of all these factors, only application rate is controllable by the operator. Slope and length of terrace are largely dependent on site characteristics. Surface microtopography is controllable to some extent by careful site preparation, but each terrace will develop different hydraulic pathways. Infiltration rate and evapotranspiration will vary from site to site depending on soil characteristics and climate conditions. Vegetation density, or surface roughness, varies with the type of vegetation and the maturity of the terrace. A mature terrace usually has a higher vegetation density and will normally have an organic mat near the soil surface caused by a buildup of grass clippings from previous harvests. This organic mat increases the resistance to flow which, in turn, increases detention time.

### Theoretical development

At a well designed and operated overland flow site, water flows downslope as a thin sheet until it freefalls into a runoff collection ditch. Under these conditions, the overland flow system operates in the laminar flow regime (Kirkby 1978) where Reynolds numbers are less than 500 (Streeter 1966). At the CRREL site Reynolds numbers ranged between 38 and 226, which is well within the laminar flow regime. For the simplest case of laminar flow over a smooth surface, the average velocity  $v_s$  can be described by the following equation (Nakano 1978):

$$v_s = \frac{g S d^2}{3 \nu} \quad (\text{m s}^{-1}) \quad (1)$$

where  $g$  = gravitational constant,  $9.81 \text{ m s}^{-2}$

$S$  = slope,  $\text{m m}^{-1}$

$d$  = average depth of flow,  $\text{m}$

$\nu$  = kinematic viscosity,  $\text{m}^2 \text{ s}^{-1}$ .

For an overland flow system, resistance to flow will be greater because of the grass and vegetative litter. Therefore, the average overland flow velocity  $V$  will be lower than the smooth surface velocity  $v_s$  and can be expressed as

$$V = a v_s \quad (\text{m s}^{-1}) \quad (2)$$

where  $a$  is a resistance coefficient. Substituting eq 2 into 1, the velocity of flow over an overland flow terrace can be calculated by

$$V = a \left[ \frac{g S d^2}{3 \nu} \right], \text{ where } a < 1.0. \quad (3)$$

If one assumes that most of the water flows in a relatively straight path downslope, the velocity  $V$  can also be expressed as

$$V = \frac{L}{\bar{t}} \quad (4)$$

where  $L$  is the length of terrace in meters, and  $\bar{t}$  the hydraulic detention time in seconds.

Also, from the continuity equation, the average depth of flow  $d$  can be determined by

$$d = \frac{Q \bar{t}}{L W} \quad (5)$$

where  $Q$  is the average overland flow rate ( $\text{m}^3 \text{ s}^{-1}$ ) and  $W$  the width of the terrace in meters.

Substituting eq 4 and 5 into eq 3 and rearranging terms, detention time can be calculated as follows:

$$\bar{t} = \left[ \frac{3 \nu W^2}{a g S Q^2} \right]^{1/3} L. \quad (6)$$

In more convenient terms with the average detention time described in minutes ( $\bar{T}$ ) and the average overland flow rate ( $q$ ) in  $\text{m}^3 \text{ hr}^{-1}$  of width, eq 6 becomes

$$\bar{T} = 5.65 \left[ \frac{\nu}{a g} \right]^{1/3} \frac{L}{S^{1/3} q^{2/3}}. \quad (7)$$

Assuming a kinetic viscosity of  $0.112 \times 10^{-5} \text{ m}^2 \text{ s}^{-1}$  (at  $15.6^\circ\text{C}$ ) and substituting the value of the gravitational constant  $g$  ( $9.81 \text{ m s}^{-1}$ ) eq 7 is reduced to

$$\bar{T} = 0.0274 \frac{L}{a^{1/3} S^{1/3} q^{2/3}}. \quad (8)$$

### Determination of resistance coefficient, $a$

To determine  $a$ , eq 8 was evaluated using detention time data obtained from the CRREL overland flow test site. For each CRREL test section, the values of  $L$  and  $S$  are 30.5 and  $0.05 \text{ m m}^{-1}$  respectively. Substituting these values, eq 8 becomes

$$\bar{T} = \frac{2.27}{a^{1/3} q^{2/3}}. \quad (9)$$

By plotting detention time vs the average overland flow rate on log-log paper,  $a$  can be determined from the line of best fit. This was done for the CRREL data shown in Figure 5. A regression analysis indicates good correlation ( $r \approx 0.78$ ) between application rate and detention time. However, the standard deviation is large, indicating that detention time varied considerably for a given overland flow rate. For example, at an application rate of  $0.2 \text{ m}^3 \text{ hr}^{-1} \text{ m}^{-1}$  of width, the predicted detention time is 34 minutes. Within one standard deviation, detention times could range from 23 to 48 minutes. Most of this deviation appears to be caused by a difference in results obtained during the 1978 and 1979 growing seasons.

The detention times were generally higher in 1979 than 1978 for the same overland flow rate. A possible explanation for this difference is an increase in vegetation density during 1979 which caused an increase in resistance to flow. This conclusion is supported by the higher grass yields in 1979 than 1978 (Palazzo in prep.). Another reason for the increased detention times could be the presence

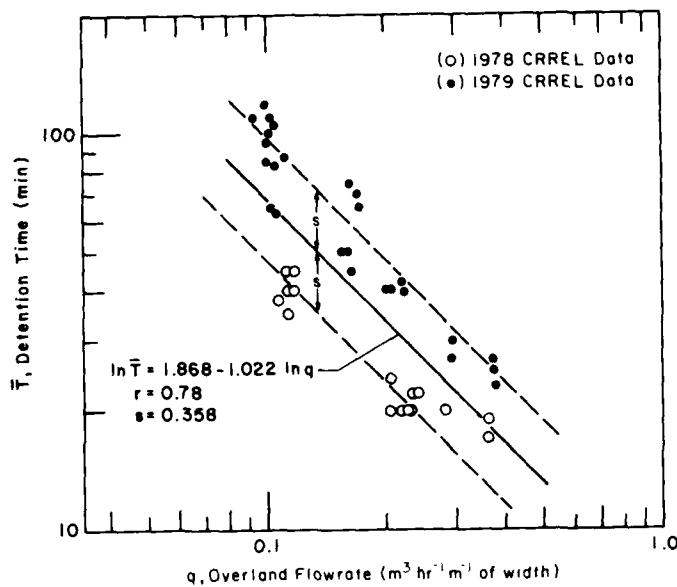


Figure 5. Overland flow rate vs detention time for CRREL overland flow test site.

of a thicker organic mat due to accumulated grass clippings from previous harvests. Tracer studies conducted during the 1980 growing season did not show a further increase in detention time. This observation suggests that the CRREL overland flow terraces reached full maturity or the maximum practical resistance coefficient after three years of operation.

The equation for the line of best fit shown in Figure 5 is

$$\ln \bar{T} = 1.868 - 1.022 \ln q \quad (10)$$

or

$$\bar{T} = \frac{6.48}{q^{1.022}}. \quad (11)$$

Substituting eq 11 for  $\bar{T}$  in eq 9, an expression for the resistance coefficient is

$$\alpha = 0.043 q^{1.066} \approx 0.043 q. \quad (12)$$

This expression indicates that the resistance coefficient  $\alpha$  increases in direct proportion to the average overland flow rate. This relationship can be explained by the fact that, as the flow rate increases, the depth of flow also increases. On the irregular surface of

most overland flow terraces, increasing the depth causes more surface area to be wetted, which increases the resistance to flow. This hypothesis is consistent with visual observations at the CRREL site and several other overland flow sites.

Substituting eq 12 back into eq 8, an empirical relationship that can be used for predicting detention time at the CRREL site is

$$\bar{T} = \frac{0.078 L}{S^{1/3} q}. \quad (13)$$

This equation indicates that  $\bar{T}$  is directly proportional to  $L$  and inversely proportional to  $q$ . Slope, being to the one-third power, is less significant although it cannot be considered negligible. For example, assuming  $L = 50$  m and  $q = 0.2 \text{ m}^3 \text{ hr}^{-1} \text{ m}^{-1}$ , an increase in slope from 2 to 12% would decrease detention time from 72 to 40 minutes, a decrease of 44%.

#### Validation

To determine the validity of eq 13 for other systems, detention times were measured at two overland flow sites. The first site, located near Utica, Mississippi, was a research facility operated by the U.S. Army Engineer Waterways Experiment Station (WES). This site (no longer in operation) had 24

terraces, each 4.5 m long x 4.5 m wide and slopes of 2, 4 and 8% (Peters et al. 1981). Detention times were measured using the same procedure developed at CRREL.

The second site is located indoors at the University of California at Davis. Each laboratory scale model terrace is 6 m long x 1.5 m wide and set at a 4% slope (Smith et al. 1980). Deionized water was used as the tracer and a response curve was developed by measuring specific conductance.

A combined total of 40 detention time measurements were taken at both sites. Measured detention times are shown in Table 3 along with the predicted detention times calculated from eq 13. The average difference between predicted and measured detention times was only 8 minutes. In most cases the measured detention time was longer than predicted, which allows an extra margin of safety in the design. In a Student's *t* distribution, the difference between measured and predicted detention time was not significant at the 95% level.

Although the average difference between predicted and measured detention time was not significant, individual differences were considerable. This is understandable, considering the variability of the surface microtopography from one terrace to another. Construction techniques, patterns of vegetative growth and harvesting operations are also factors which can change the hydraulic detention time.

## KINETICS

Kinetic relationships describing removal of BOD, TSS,  $\text{NH}_3\text{-N}$  and total P were developed by taking several detention time measurements during each application period. The average detention time ( $\bar{T}$ ) was then calculated along with the average percent removal on a mass basis for each constituent. All raw data used in this development can be found in Appendix A.

### BOD removal

BOD is removed by sedimentation, filtration and biological oxidation (U.S. EPA 1977). The first two mechanisms are responsible for removing particulate BOD. The soluble BOD is oxidized by microorganisms which are probably similar to the attached biomass found in trickling filters. However, some soluble organic compounds are released from the plant-soil system, and as a result, runoff BOD concentrations below 3 to 5 mg/L cannot be expected (Overcash et al. 1976).

Temperature also has an effect on runoff BOD concentrations. Martel et al. (1980) found that BOD concentrations in the runoff exceeded 30 mg/L at soil temperatures at or below 4°C. However, temperature effects should not be a significant problem at full-scale facilities if wastewater is stored during the winter. In this study, temperature effects were nullified by selecting performance data obtained during the growing season only (April through October).

The experimental data obtained at CRREL and the University of California, Davis (Fig. 6) indicate that BOD removal can be expressed as a first-order equation in the form

$$\text{Percent removal} = (1 - A e^{-k \bar{T}}) 100. \quad (14)$$

The coefficients  $A$  and  $k$ , obtained by a least-squares fit to the data, were 0.52 and  $0.03 \text{ min}^{-1}$  respectively. The coefficient  $k$  is the average kinetic rate constant. The coefficient  $A$  can be interpreted as the non-settleable fraction of BOD in the applied wastewater while the remaining settleable fraction (0.48) is removed during the first few meters or minutes after wastewater is applied. This conclusion is supported by the BOD vs downslope distance data shown in Figure 7 where 44% of the BOD was removed within the first 5 m.

### TSS removal

Total suspended solid (TSS) removal vs average detention time from the CRREL site is shown in Figure 8. The flat slope of the estimated line of best fit indicates that TSS removal changed little over the range of detention times tested. For example, at a detention time of 20 minutes, TSS removal was 86%. A three-fold increase in detention time (60 min) only increased removal by 6%.

The high solids removal efficiency of the overland flow process is due to the shallow depth of water and the long travel distance to the end of the terrace. Even minute particles with slow settling velocities are able to settle out before reaching the collection ditch. Also, grass and vegetative litter help to entrap and filter out particles. Data plotted in Figure 7 indicate that most of the suspended solids were removed within the first 5 m.

Because of rapid settling, a buildup of solids is apparent at the top of most overland flow terraces which receive raw or primary wastewater. At the CRREL site, solids deposition was heavy enough in some spots to smother grass growth. Similar condi-

**Table 3. Measured and predicted detention times.**

Location and date	Application rate ( $m^3 hr^{-1}$ )	Average* overland flow rate ( $m^3 hr^{-1} m^{-1}$ )	Slope (%)	Measured detention time, $\bar{T}_p$ (mm)	Predicted detention time, $\bar{T}_p$ (min)	$\bar{T}_p - \bar{T}_m$ (min)
<i>Utica, Miss.</i>						
13 July 78	0.584	0.097	4	75	108	33
14 July 78	0.584	0.097	4	60	108	43
21 July 78	0.584	0.097	4	75	108	33
21 July 78	0.584	0.097	4	90	108	18
20 July 78	0.435	0.072	4	130	146	16
2 Aug 78	0.292	0.049	8	255	170	-85
2 Aug 78	0.292	0.049	8	255	170	-85
3 Aug 78	0.435	0.072	8	150	116	-34
3 Aug 78	0.435	0.072	8	165	116	-49
17 Aug 78	0.435	0.072	8	120	116	-4
17 Aug 78	0.435	0.072	8	135	116	-19
17 Aug 78	0.435	0.072	2	165	184	19
17 Aug 78	0.435	0.072	2	150	184	34
31 Jan 79	0.435	0.072	2	135	157	22
31 Jan 79	0.435	0.072	2	150	157	7
8 Feb 79	0.435	0.072	2	165	157	-8
25 Jan 79	0.435	0.072	8	120	99	-21
2 Feb 79	0.435	0.072	8	105	99	-6
2 Feb 79	0.435	0.072	8	165	99	-66
10 Feb 79	0.435	0.072	8	96	99	3
10 Feb 79	0.435	0.072	8	135	99	-36
24 Jan 79	0.584	0.113	4	105	93	-12
15 May 79	1.753	0.292	2	90	45	-45
22 May 79	0.435	0.072	2	330	184	-146
Aug 79	1.753	0.292	4	50	36	-14
Aug 79	0.876	0.146	4	135	72	-63
Aug 79	1.753	0.292	4	65	36	-29
Aug 79	1.753	0.292	8	48	29	-19
11 Feb 80	0.219	0.042	2	207	314	107
12 Feb 80	0.435	0.084	2	112	157	45
12 Feb 80	0.435	0.084	2	120	157	37
12 Feb 80	0.435	0.084	8	90	98	8
12 Feb 80	0.435	0.084	8	75	98	23
13 Feb 80	0.876	0.169	2	75	78	3
13 Feb 80	0.876	0.169	2	75	78	3
13 Feb 80	0.876	0.169	8	60	49	-11
13 Feb 80	0.876	0.169	8	60	49	-11
<i>U. of Calif., Davis</i>						
	0.118	0.079	4	21	17	-3
	0.236	0.157	4	13	9	-4
	0.345	0.236	4	11	6	-5
					<i>n</i>	40
					mean	-8
					std. dev.	43

\*At Utica, 0.75 and 0.87 were used as overland flow coefficients for summer and winter respectively. A runoff coefficient of 1.0 was used at U. of Calif., Davis.

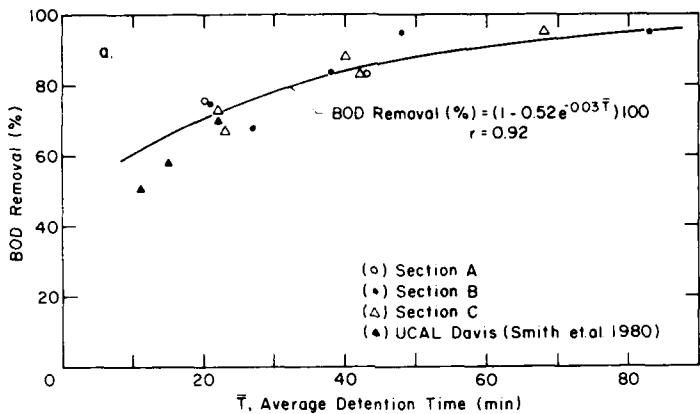


Figure 6. BOD removal efficiency vs detention time.

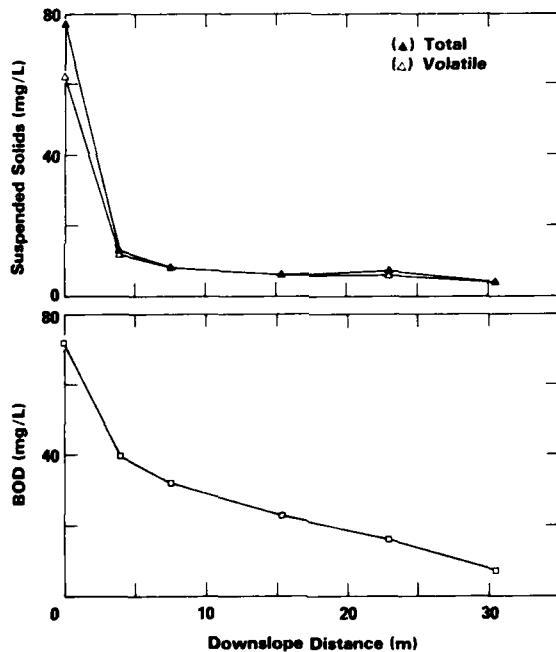


Figure 7. Concentration of BOD and suspended solids vs downslope distance on section C (25 June 1980).

tions were observed during a visit at the Easley, South Carolina, site. After years of application, solids buildup can be substantial and can cause an odor problem if these deposits are allowed to become anaerobic (Scott and Fulton 1979). However, this problem can be avoided by occasionally disk-in the solids and allowing enough reaeration time after each application. Disking should be done on only

part of the terraces so that the rest of the system remains in service.

The solids removal relationship developed in this study (Fig. 8) applies to fecal types of solids only. Removal of algal solids found in lagoon effluent is more difficult to predict. Data from the Easley, South Carolina, site indicated that algae removal by overland flow is marginal (Pollock 1979). However,

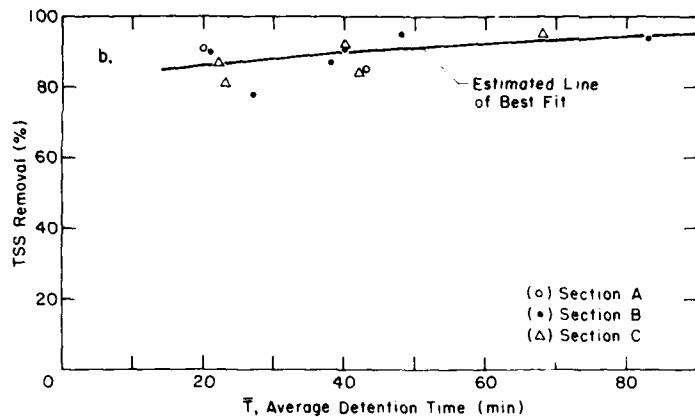


Figure 8. Total suspended solids removal efficiency vs detention time.

Peters et al. (1980) report removal of algae at low application rates.

#### Nitrogen removal

A number of mechanisms are involved in nitrogen removal, including volatilization, nitrification-denitrification, adsorption, plant uptake and soil storage. The ammonia form of nitrogen can be removed by any of the above mechanisms. Most of the organic nitrogen is initially removed by sedimentation and then incorporated into the soil or converted to ammonia by saprophytic bacteria. Nitrate is the most difficult form of nitrogen to remove (Jenkins et al. 1978, Walter 1974). Nitrate ions have little affinity for soil particles and thus are not retained on the overland flow terrace. For efficient removal

of nitrogen, raw wastewater or primary effluent should be applied because these wastewaters contain very little nitrate.

This study focused on the kinetics of ammonia removal because it is the nitrogen form of most concern in discharge limitations. The correlation between ammonia removal and detention time obtained from CRREL data is shown in Figure 9. The first-order equation which closely fits these data ( $r = 0.91$ ) is also shown in Figure 9. For ammonia removal the coefficients  $A$  and  $k$  were 0.81 and  $0.03 \text{ min}^{-1}$  respectively.

It is interesting to note that both BOD and ammonia removal equations (see Fig. 6 and 9) contain the same kinetic rate constant ( $k = 0.03 \text{ min}^{-1}$ ), suggesting that both BOD and ammonia removal are controlled

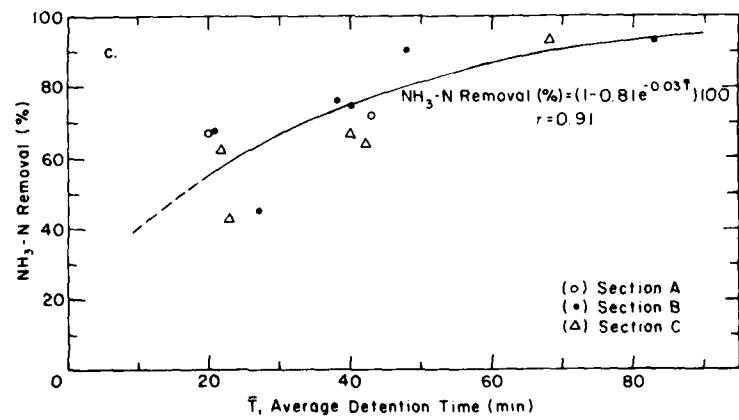


Figure 9. NH<sub>3</sub>-N removal efficiency vs detention time.

by the same rate-limiting step. It is unlikely that both substrates would have the same removal rate constant; a more likely explanation is that removal rate is mass transport limited. In other words, the rate of mass transport from the bulk liquid to the active biomass and adsorption sites is the mechanism governing removal rate. This reasoning is reinforced by the fact that overland flow operates in a laminar flow regime, which reduces the opportunities for substrate contact with reactive sites.

#### Phosphorus removal

According to the *Process Design Manual for Land Treatment of Municipal Wastewater* (U.S. EPA 1977) phosphorus is removed primarily by sorption to soil particles. On overland flow terraces only surface exchange sites are available because most of the wastewater passes over the soil surface rather than through it. As a result, the exchange sites are used up rather quickly and the removal of phosphorus by overland flow systems is limited. Plant uptake is another mechanism capable of removing phosphorus. Palazzo et al. (1980) reported that forage grasses removed 54% of the applied phosphorus at the CRREL site.

As shown in Figure 10, our studies indicated that phosphorus removal did not change significantly over the range of detention times tested. Percentage removals ranged between 37 and 61% and averaged 53%. Analyses of runoff samples indicated that most of the total phosphorus was in the "ortho" form, which indicates that the phosphorus removed was tied up with particulate matter. As discussed earlier (see *TSS Removal*), particulate matter was easily removed by overland flow.

Phosphorus removal can be improved by adding alum to the wastewater prior to application. Thomas et al. (1976) increased phosphorus removal to 90% using this technique. Similar results were obtained at the Utica, Mississippi, overland flow site (Peters et al. 1981).

#### Validation

The kinetic relationships for removal of BOD, TSS and  $\text{NH}_3\text{-N}$  were validated by comparing the predicted removal to the actual removal reported at seven full-scale systems. Statistical analysis of these data indicated that the average differences between predicted and actual BOD, TSS and  $\text{NH}_3\text{-N}$  removal were only 1.9, -2.0 and 2.8%, respectively, for systems receiving primary or raw wastewater (see Table 4). However, when systems receiving pond effluent were evaluated, the predicted removals for BOD and TSS were 18 and 22% higher than actual (see Table 5). Higher predicted removals can be explained by the fact that pollutants remaining in pond effluent are generally less degradable, and thus more difficult to remove, than those in primary or raw wastewater. Also, there is a lower limit to the BOD and TSS concentration in the runoff. As discussed earlier, this limit is approximately  $5.0 \text{ mg L}^{-1}$ . Therefore, high removal efficiencies become more difficult to achieve as pollutant concentrations in the applied wastewater decrease.

The ammonia removal relationship (Fig. 9) appears to be valid whether primary or pond effluent is applied. The average differences between predicted and actual  $\text{NH}_3\text{-N}$  removal were only 2.8 and -4.5% for systems receiving primary and pond effluent, respectively.

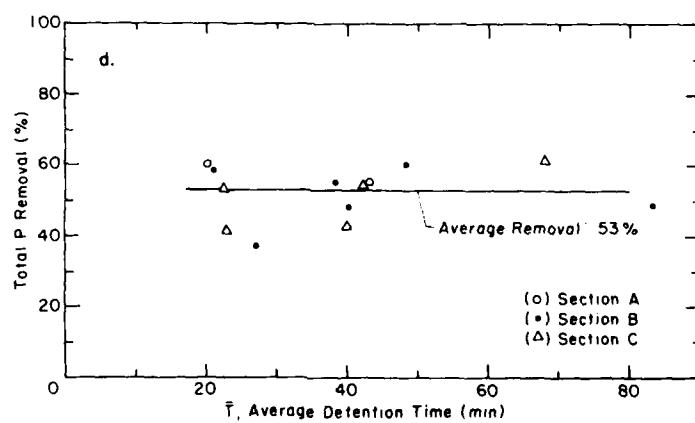


Figure 10. Total phosphorus removal vs detention time.

**Table 4. Predicted vs actual removal efficiencies on overland flow systems receiving primary or raw wastewater.**

System	Calculated detention time (min)	BOD				TSS				NH <sub>3</sub> -N			
		Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted removal (%)	Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted removal (%)	Applied conc. (mg/L)	Predicted removal (%)	Actual removal (%)	Predicted removal (%)
Ada, Oklahoma (Thomas et al. 1976)	222 195 171	0.50 0.50 0.50	150 99+ 150	96 97 97	3 2 2	160 95+ 160	97 98 98	97 -3 -3	-2 -3 -3	17.0 17.0 17.0	99+ 99+ 99+	98 96 97	1 3 2
Pauls Valley, Oklahoma (Hall et al. 1977)	294	0.50	117	99+	96	3	105	95+	97	-2	17.0	98	91
Werribee Farm, Australia (Scott and Fulton 1979)	626	0.80	507	99+	98	3	233	95+	93	2	31.0	99+	20*
Easley, South Carolina (Pollock, 1979)	59	0.70	200	91	91	0	186	92	97	-5	19.4	86	85
Paris, Texas (C.W. Thornthwaite, 1979)	138	0.60	480	99	99	0	181	95+	96	-1	-	-	-
Mean										1.9	-2.0	2.8	
Std. dev.										1.3	2.2	2.5	

\*Wastewater was applied during the winter when crops were not actively growing.

**Table 5. Predicted vs actual removal efficiencies on overland flow systems receiving pond effluent.**

System	Calculated detention time (min)	Runoff fraction	BOD			TSS			NH <sub>3</sub> -N			
			Predicted		Actual removal (%)	Predicted		Actual removal (%)	Predicted		Actual removal (%)	
			Applied conc. (mg/L)	Predicted conc. (mg/L)	- actual removal (%)	Applied conc. (mg/L)	Predicted conc. (mg/L)	- actual removal (%)	Applied conc. (mg/L)	Predicted conc. (mg/L)	- actual removal (%)	
Utica, Mississippi (Peters et al. 1981)	354	0.80	22	99+	4	30	95+	93	2	3.5	99	0
	91	0.80	22	97	23	30	95+	71	24	3.5	95	0
Easley, South Carolina (Pollock 1979)	56	0.70	28	90	63	27	60	92	53	39	1.0	85
Carbondale, Illinois (Himrich et al. 1980)	17	0.83	27	69	40	29	22	86	—	—	4.5	51
	17	0.83	18	69	73	4	24	86	—	—	8.0	51
	17	0.83	70	69	85	-16	35	86	72	14	31.6	51
	24	0.83	44	64	68	-4	34	84	51	33	29.6	43
	24	0.83	9	75	38	37	—	—	—	—	13.6	60
	24	0.83	15	75	36	39	26	87	—	—	9.8	60
	24	0.83	20	75	31	44	24	87	65	22	16.8	60
Mean							17.9			22.3		-4.5
Std. dev.							21.1			13.2		11.6

The results of this analysis indicate that the kinetic relationships for BOD and TSS removal are valid for overland flow systems receiving primary or raw wastewater only. The lowest applied concentration of BOD and TSS where these relationships hold is estimated to be 45 mg L<sup>-1</sup>. Different kinetic relationships need to be developed for overland flow systems receiving pond effluent.

## DESIGN PROCEDURE

The primary purpose of the proposed design procedure is to properly size the overland flow system so that it meets the quality requirements of the discharge permit. The three basic steps in this procedure are

1. Determine the detention times required to remove pollutants specified in the discharge permit.
2. Calculate the application rate needed to satisfy the longest or most critical detention time.
3. Calculate the land area required from the application rate and system design flow.

### Step 1: Determine the detention time

The detention time required to achieve the desired removal of BOD, TSS and NH<sub>3</sub>-N can be determined from Figures 6, 8, and 9. The phosphorus removal vs detention time relationship shown in Figure 10 is not used in this procedure because overland flow is not considered to be an efficient process for removing this pollutant. As noted earlier, phosphorus removal can be improved by chemical precipitation with alum.

To use Figures 6, 8, and 9, the percentage removal of BOD, TSS and NH<sub>3</sub>-N must be calculated on a mass basis. Information needed for this calculation includes the design flow, an estimate of the applied wastewater concentrations, desired runoff concentrations and the runoff fraction.

Concentrations of BOD, TSS and NH<sub>3</sub>-N in the applied wastewater will depend on the degree of pretreatment. In most cases it is advisable to use less pretreatment in order to reduce costs and take advantage of the excellent treatment capabilities of overland flow. If the design includes a storage pond, the diluting effect of this effluent when mixed with primary or raw wastewater should also be considered.

The desired runoff concentrations can be determined from the discharge permit. It should be noted that by satisfying the concentration limits specified

in the discharge permit, the mass of pollutant discharged will be smaller than in conventional wastewater treatment systems. As explained earlier, less wastewater is discharged from overland flow systems than is applied.

The fraction of wastewater that reaches the runoff collection ditch can be estimated from local evapotranspiration and percolation rates. Typically, the runoff fraction will range from 0.6 to 0.9 depending on local climatic conditions and soil characteristics.

The detention time used for design should be the longest time determined from Figures 6, 8, and 9. For equal removal percentages, the controlling design parameter is ammonia removal followed by BOD and TSS removal. For example, if 90% ammonia, BOD and TSS removal is required, the detention times needed are 68, 57 and 40 minutes respectively. In this case, the design should be based on ammonia removal since it requires the longest detention time. However, in most cases ammonia removal is not required and BOD removal will be the controlling design parameter.

### Step 2: Calculate the application rate

The annual application rate which will satisfy the detention time requirements determined in Step 1 can be calculated as follows:

$$Q_a = \frac{q}{r} Y \quad (15)$$

where  $Q_a$  = annual application rate, m<sup>3</sup> yr<sup>-1</sup> m<sup>-1</sup>  
 $r$  = overland flow coefficient  
 $Y$  = operating time, hr yr<sup>-1</sup>.

The average overland flow rate  $q$  can be calculated by rearranging eq 13 so that

$$q = \frac{0.078 L}{S^{1/3} \bar{T}} \quad \text{where } q \leq \frac{0.2}{S^{1/3}} \quad (16)$$

The values for  $L$  and  $S$  are selected by the designer based on the topography of the potential site. Natural terrain contours should be used to the maximum extent possible to minimize cut and fill operations. However, careful surface preparation will still be needed to ensure even flow distribution.

The upper limit on  $q$  in eq 16 is necessary to avoid a situation where the application rate is so high that it causes scouring. Typically this could occur when a design calls for a short detention time on a long terrace. The limitation placed on  $q$  was based on calculations of the scour velocity shown in Appendix C.

The overland flow coefficient  $r$  is the average fraction of applied wastewater flowing over the soil surface. The purpose of this coefficient is to convert the average overland flow rate  $q$  to an application rate ( $Q_a$ ). It can be calculated from the relationship

$$r = \frac{1.0 + f}{2} \quad (17)$$

where  $f$  is the runoff fraction.

The annual operating time  $Y$  will vary depending on the application cycle and season. The application cycle is the number of hours per day and days per week that wastewater is being applied to the terraces. Application cycles normally range from 6 to 8 hours of continuous application per day, 5 to 7 days per week. Obviously, the land area required for the system can be reduced substantially by choosing a longer application cycle. No deleterious effects on performance were noted at Utica, Mississippi (Peters et al. 1981) when pond effluent was applied 24 hours per day, 7 days per week. However, shorter application cycles of 8 to 10 hours per day are recommended if raw wastewater or primary effluent is applied. Shorter application cycles will reduce the rate of solids accumulation at the top of the terrace and allow these solids to degrade more rapidly because of aeration during the off period.

The application season is the number of weeks per year that the system can be expected to operate. In southern areas the application season may extend throughout the year because short periods of cold weather will normally not affect performance (Aly et al. 1979). In northern areas, the application season usually coincides with the growing season. During the non-growing season, wastewater is stored in a pond or lagoon. Martel et al. (1980) found that the EPA-1 computer program provided a good estimate of the number of storage days needed for overland flow systems.

#### Step 3: Calculate the land area

Since the length of terrace has already been specified, the only remaining dimension needed to calculate the land area is width. Width can easily be determined by dividing the annual volume of wastewater applied by the annual application rate  $Q_a$ . If a storage pond is included in the design, the annual volume of wastewater applied should be adjusted to reflect the net volume of water entering or leaving the pond due to precipitation and evaporation.

The land area calculated by this procedure is only part of the total wetted area. Additional wetted

area will be needed to handle wastewater flows during harvest operations. Each terrace should be harvested on a rotating basis. The length of the drying period before harvest will depend on local climatic conditions and should be long enough to allow harvesting machinery to drive over the terrace without rutting the surface. In most cases, a week should be adequate. An alternative to increasing the size of the wetted area would be to temporarily increase the application time to the remaining terraces.

Additional land will also be needed for buildings, access roads and buffer zones. Aly et al. (1979) indicated that this additional non-wetted area is usually 25% to 30% of the sum of wetted and non-wetted areas. The total area of the overland flow system should also include the land needed for a storage pond if necessary.

#### DESIGN EXAMPLE

The previously outlined design procedure can best be explained with an example. In this example, an overland flow facility is being considered for a small town in upstate New York. The design flow is  $3785 \text{ m}^3 \text{ day}^{-1}$  (1.0 million gal. day $^{-1}$ ). Because of its northern location the facility will have a holding pond to store wastewater during the winter. The holding pond effluent and raw wastewater will be mixed prior to application. The expected quality of this mixture is  $150 \text{ mg L}^{-1}$  BOD and  $100 \text{ mg L}^{-1}$  TSS. The discharge limits for this facility are  $20 \text{ mg L}^{-1}$  BOD and  $20 \text{ mg L}^{-1}$  TSS. The soils in the area are slowly permeable so that the runoff fraction is expected to be only 0.6. The mass BOD and TSS removals required to meet the discharge limits are

$$\% \text{ BOD removal} = \frac{(1.0 \times 150 - 0.6 \times 20) 100}{1.0 \times 150}$$

$$= 92\%$$

$$\% \text{ TSS removal} = \frac{(1.0 \times 100 - 0.6 \times 20) 100}{1.0 \times 100}$$

$$= 88\%.$$

From Figure 6, the detention time needed to remove 92% of the BOD is 60 minutes. From Figure 8, the detention time needed to remove 88% of the TSS is 40 minutes. Since BOD removal is the limiting parameter, the design is based on a detention time of 60 minutes.

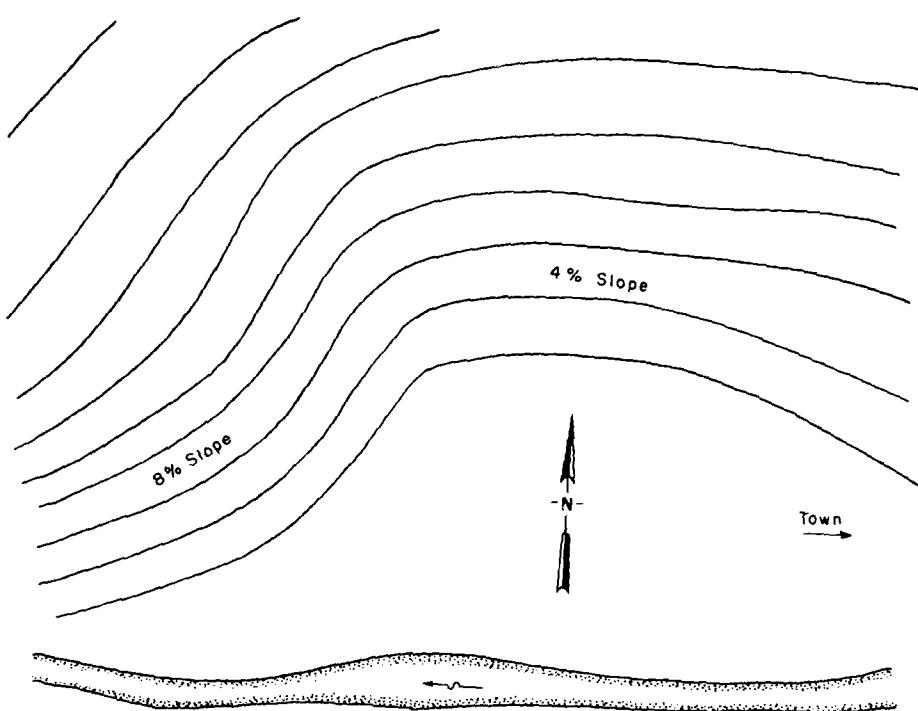


Figure 11. Topographic sketch of proposed overland flow site.

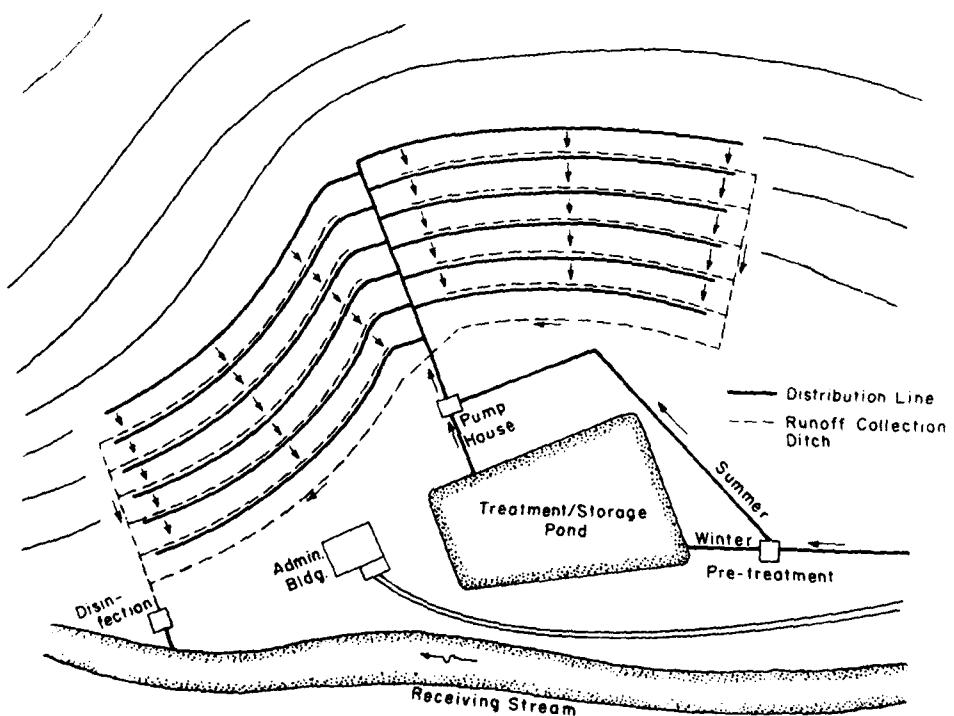


Figure 12. Possible configuration of terraces and storage pond for hypothetical overland flow system.

The proposed site for the overland flow system is on a hillside next to a river. As shown in Figure 11, the eastern half of the site has a general slope of 4% while the slope of the western half is 8%. The length of each terrace is assumed to be 50 m (160 ft). From eq 16,

$$q = \frac{0.078 (50)}{(0.04)^{1/3} 60}$$

$$= 0.190 \text{ m}^3 \text{ hr}^{-1} \text{ for the 4\% slope}$$

and

$$q = \frac{0.078 (50)}{(0.08)^{1/3} 60}$$

$$= 0.151 \text{ m}^3 \text{ hr}^{-1} \text{ m}^{-1} \text{ for the 8\% slope.}$$

With an application season of 28 weeks per year (April through October) and an application cycle of 10 hours daily for 5 days per week, the annual operating time is 1400 hrs. Also, the runoff coefficient calculated from eq 17 is 0.8. The annual application rate  $Q_a$  to each terrace can now be calculated from eq 15:

$$Q_a = \left( \frac{0.190}{0.8} \right) 1400$$

$$= 332.5 \text{ m}^3 \text{ yr}^{-1} \text{ m}^{-1} \text{ for the 4\% slope}$$

$$Q_a = \left( \frac{0.151}{0.8} \right) 1400$$

$$= 264.3 \text{ m}^3 \text{ yr}^{-1} \text{ m}^{-1} \text{ for the 8\% slope.}$$

If the annual design flow is  $1.38 \times 10^6 \text{ m}^3$  and the precipitation minus evaporation volume from the holding pond is  $0.15 \times 10^6 \text{ m}^3$ , the total water volume applied to the overland flow site is  $1.53 \times 10^6 \text{ m}^3 \text{ yr}^{-1}$ . If this volume is divided equally, the wastewater applied to each half of the site is  $0.77 \times 10^6 \text{ m}^3 \text{ yr}^{-1}$ . With an annual application rate of  $332.5 \text{ m}^3 \text{ yr}^{-1} \text{ m}^{-1}$ , the total width of terrace needed on the 4% slope is 2,316 m. Likewise, the total width of terrace needed on the 8% slope is 2,913 m. If the length of each terrace is 50 m then the combined wetted area needed for treatment is 27 ha (67 acres).

Additional wetted area will be needed to handle wastewater flows during harvest operations. If three harvests per year and a drying period of one week per harvest are planned, the wetted area should be increased by 11% (3 wks/28 wks). Therefore the

adjusted wetted area is 30 ha (74 acres). The non-wetted area needed for buildings, access roads and buffer zones would add another 10 ha based on the assumption that the wetted area is 75% of the sum of wetted and non-wetted areas (Aly et al. 1979).

The area needed for a storage pond is estimated to be 20.0 ha. This estimate is based on a storage volume of  $667,750 \text{ m}^3$  which includes  $567,750 \text{ m}^3$  for storing wastewater and  $100,000 \text{ m}^3$  for storing precipitation during winter. The storage needed for wastewater was calculated by multiplying the design flow ( $3785 \text{ m}^3 \text{ day}^{-1}$ ) by the number of storage days (150 days, November thru March). The depth of the storage pond was assumed to be 3.5 m. Summing the wetted, non-wetted and storage pond areas, the total area of the overland flow facility is 60 ha (148 acres). A possible configuration of the terraces for this overland flow system is shown in Figure 12.

#### COMPARISON WITH THE TRADITIONAL DESIGN APPROACH

As indicated earlier in this report the traditional design procedure is to calculate the wetted area based solely on hydraulic loading rate. For an overland flow system receiving primary or pond effluent, the procedure is to select a hydraulic loading ranging from 6.4 to 20 cm/wk. Using the same design flow and application season as in the previous design example, the required wetted area would range from 27 to 85 ha. Recall that the wetted area predicted by the rational procedure presented in this report was 27 ha, which is the least amount of land predicted by the traditional approach. Therefore a system designed according to the traditional procedure will require more land, especially if the designer selects a conservative loading rate. For example, it would be reasonable to assume that the designer would choose a loading rate in the middle of the range. For a hydraulic loading rate of 15 cm/wk, the land area required would be approximately 40 ha or 50% greater than that required by the rational procedure. In this case the cost of a traditionally designed system would be about 50% higher.

Beyond the monetary benefits just described, the rational procedure presented here is based on the fundamental process design concept of detention time. This concept is familiar to most designers, which makes overland flow more appealing as a treatment alternative. Also, this procedure allows the designer to tailor each site according to existing site conditions and discharge requirements.

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### APPENDIX A: CRREL OVERLAND FLOW DATA

#### Applied and Runoff Water Quality from Section A of CRREL Overland Flow Test Site.<sup>1</sup>

Date	Appl. m <sup>3</sup> /hr/m	Appl. mm	Detention time min.	St. volume on water surface, <i>f</i>	BOD mg/L	TSS mg/L	TKN mg/L	NO <sub>x</sub> -N mg/L	NO <sub>3</sub> -N mg/L	Tot. P mg/L	PO <sub>4</sub> mg/L	Turb. NTU	Appl. Runoff m <sup>3</sup> /hr/m	Appl. Runoff mm	Appl. Runoff m <sup>3</sup>	Appl. Runoff m <sup>3</sup> /hr	Appl. Runoff m <sup>3</sup> /100 ml.	Appl. Runoff m <sup>3</sup> /100 ml.							
24 Aug '88	0.112	45	9.92	688	109	31	0	4	43	18	34	14	0.0	4.8	7.8	5.5	4.3	11	7.2	7.1	603	568	52		
25 Aug	0.111	1860	1348	158	32	28	10	51	21	34	16	0.0	2.0	7.5	5.4	4.7	13	5.9	7.1	7.3	693	662			
26 Aug					41	12	44	21	31	16	0.0	3.2	9.0	5.3	6.6	5.1	18	7	7.3	7.4	687	655			
30 Aug	0.109	1402	901	57	15	28	13	0.0	2.3	28	13	0.0	2.3	28	6	7.2	7.2	687	655						
1 Sept	0.114	40	1560	1079	66	15	51	14	36	17	0.0	2.3	29	6	7.5	7.4	35	19	7.1	6.8	589	484			
5 Sept	0.097	1170	662	75	22	30	9	0.0	4.3	11	0.0	3.4	10.4	5.5	7.6	5.4	6.3	6	7.4	7.5	574	532	42		
6 Sept	0.114	45	992	659	72	21	66	9	49	16	31	11	0.0	3.4	10.4	5.5	7.6	5.4	34	8	7.4	7.3	556	562	
14 Sept	0.216	20	1020	751	45	14	91	7	-	31	17	0.1	1.0	31	13	0.0	1.9	8.7	5.1	5.0	9	7.0	547	570	
15 Sept	0.201	20	1260	803	72	24	93	13	-	31	13	0.0	1.9	8.7	5.1	6.5	4.3	-	-	-	-	-	-		
19 Sept	0.222	1440	1139	107	9	41	15	28	11	0.0	1.4	8.0	4.4	4.9	9	7.5	7.4	565	452						
20 Sept	0.221	1140	893	42	16	112	5	-	34	14	0.0	2.1	35	7	7.1	7.2	582	464	34	30					
21 Sept	0.218	20	1260	954	66	17	71	7	32	14	0.0	1.8	26	4	7.2	7.3	572	561							
25 Sept	0.195	840	437	64	39	22	31	14	0.0	2.5	10.5	7.4	9.1	7.0	27	10	7.2	7.0	597	490					
28 Sept	0.216	1020	754	51	18	52	13	19	14	0.2	1.8	-	-	18	9	7.1	7.0	436	352						
29 Sept	0.216	20	1080	795	62	20	79	16	-	24	7	1.2	1.8	29	8	7.1	7.2	503	528						
2 Oct	0.202	1050	776	22	127	15	32	10	0.0	1.6	-	-	45	10	7.4	7.4	545	449							
3 Oct	0.201	1260	716	141	19	41	15	34	13	0.0	1.1	9.7	5.9	5.7	4.7	42	8	7.1	7.1	554	442				
4 Oct	0.209	-	960	656	107	39	117	22	-	30	13	0.0	1.1	36	8	7.3	7.1	518	417	40					
5 Oct	0.220	5140	2729	83	27	92	8	36	20	25	15	0.2	0.7	5.3	3.2	3.3	5	7.0	7.1	485	421				
12 Oct	0.211	3600	2502	89	33	140	24	47	33	33	17	0.0	1.3	9.1	4.6	7.7	4.3	6.3	10	7.2	7.2	621	508		
27 Oct	0.359	17	4900	3596	59	28	88	20	-	32	22	0.0	0.9	5.0	12	7.4	7.4	764	721						
30 Oct	0.369	2400	1889	93	18	44	30	32	25	0.3	1.8	6.4	4.7	4.7	3.3	8.0	4.0	7.5	7.3	707	541				

<sup>1</sup> Because of leaks in the membrane, application to Section A was discontinued during 1979.

## Applied and Runoff Water Quality from Section B of CRRREL Overland Flow Test Site:

**Applied and Runoff Water Quality from Section B of CRREL Overland Flow Test Site (cont'd).**

Date	Average overland flow rate, $q$ (m <sup>3</sup> /hr/m)	Storage, state volume of wastewater, $L$ (m <sup>3</sup> )	Retention time (min)	Applied Runoff (mm)	SO <sub>4</sub> <sup>2-</sup> (mg/L)	TSS (mg/L)	IA (mg/L)	NO <sub>3</sub> -N (mg/L)	NO <sub>2</sub> -N (mg/L)	Intr. P (mg/L)	PO <sub>4</sub> <sup>3-</sup> (mg/L)	Turb. (NTU)	Specific conduct. (mhos/cm)	pH	Temp. (°C)	Applied Runoff (mm)							
29 Jun 79	0.110	-	1,761	1253	28	2	-	2	1.2	2.2	1.6	3.6	12	2	7.2	7.0	375	296	28	4.2 x 10 <sup>4</sup>	6.5 x 10 <sup>4</sup>		
2 Jul	0.117	-	1878	1646	40	2	-	18	3	0.1	1.3	26	5	7.2	7.2	-	-	-	-	-	-		
3 Jul	0.099	-	2160	1302	40	3	-	21	2	0.1	1.5	24	3	7.3	7.2	-	-	-	-	-	-		
5 Jul	0.097	-	1776	1015	36	4	-	22	2	0.1	3.5	2.7	2.7	19	4	7.1	6.9	397	267	-	-		
6 Jul	0.101	-	2118	1342	29	4	19	3	21	2	0.0	3.5	12	4	7.3	7.0	-	-	-	-	-	-	
9 Jul	0.102	-	1254	807	43	4	-	22	2	0.0	2.0	-	-	28	1	7.2	6.9	-	-	-	-		
10 Jul	0.109	-	1896	1432	29	2	-	19	1	0.0	1.5	-	-	17	3	7.2	7.0	-	-	-	-		
11 Jul	0.098	-	1812	1071	48	3	34	2	18	0	0.0	1.2	-	-	24	3	7.2	7.1	-	-	-	-	
12 Jul	0.108	-	1800	1331	53	2	45	1	24	3	0.0	1.5	-	-	25	2	7.4	7.4	386	256	-	-	
13 Jul	0.108	-	1912	1410	55	3	49	2	28	4	21	2	0.0	1.7	4.0	2.5	3.7	2.7	22	2	7.4	7.2	
16 Jul	0.107	-	1566	1135	49	8	-	16	2	0.0	2.4	-	-	24	6	7.1	7.0	-	-	-	-		
17 Jul	0.108	-	1794	1318	43	4	-	17	0	0.0	1.3	-	-	24	4	7.2	7.2	-	-	-	-		
18 Jul	0.095	-	1728	55	63	-	20	16	0.0	-	5.8	4.9	28	5	7.1	7.1	-	-	-	-	-	-	
19 Jul	0.105	-	1800	1218	51	4	45	4	17	1	0.0	1.5	-	-	24	3	7.2	7.2	-	-	-	-	
20 Jul	0.101	130	1800	1138	28	3	-	14	0	0.0	1.4	-	-	18	3	7.3	7.1	-	-	-	-		
27 Jul*	0.105	-	1290	882	36	26	43	10	27	9	16	3	0.0	9.4	4.4	5.5	4.0	4.8	23	8	6.9	562	
30 Jul	0.095	-	1254	669	37	6	22	8	15	2	0.2	9.5	4.1	4.5	25	5	7.1	6.9	569	30	103	3.3 x 10 <sup>4</sup>	
31 Jul	0.091	-	1806	1806	33	-	-	15	15	0.2	-	-	-	17	1	7.1	7.1	-	-	-	-		
1 Aug	0.100	85	1806	1124	40	4	61	3	16	1	0.1	3.0	-	-	22	2	7.2	7.2	-	-	-	-	
2 Aug	0.174	-	2619	2266	59	4	26	3	-	16	1	0.0	2.7	-	-	15	3	7.2	7.2	-	-		
3 Aug	0.173	-	2760	2345	47	4	23	2	-	15	1	0.2	3.3	-	-	17	2	7.3	7.2	-	-		
6 Aug	0.160	-	2109	1589	29	4	-	-	17	1	0.1	3.5	-	-	21	6	7.3	7.0	-	-	-	-	
7 Aug	0.160	50	2601	1884	36	4	31	5	23	2	0.0	3.3	4.4	2.6	2.7	1.9	24	4	7.4	7.0	4.78	301	
8 Aug	0.154	-	1593	1045	88	5	82	3	-	20	1	0.0	2.8	-	-	46	3	7.3	7.0	-	-		
9 Aug	0.144	-	29.0	30.0	48	36	-	-	18	-	0.0	-	-	-	20	-	-	-	-	-	-		
13 Aug	0.134	-	3054	-	77	-	-	-	16	1	0.0	1.0	-	-	49	-	-	-	-	-	-		
14 Aug	0.166	-	3168	2596	51	2	-	16	1	0.0	1.8	-	-	26	2	6.9	7.0	303	225	20	17		
15 Aug	0.164	-	2700	2027	64	2	74	2	21	3	0.0	2.0	3.9	2.2	2.9	2.5	28	2	7.3	7.0	-	-	
16 Aug	1.162	-	2160	1609	61	3	50	2	-	14	1	0.0	2.0	-	-	27	2	7.3	7.0	-	-		
17 Aug	0.169	-	3312	2582	85	4	60	3	-	25	3	0.0	5.0	-	-	40	3	7.4	7.1	-	29		
20 Aug	0.174	-	3456	2582	128	2	-	-	25	5	0.0	5.1	-	-	47	6	7.4	7.2	-	31			
21 Aug	0.176	-	3437	2651	44	3	-	-	21	5	0.0	1.7	-	-	22	4	7.3	7.2	-	28			
22 Aug	0.177	-	2208	1720	76	7	45	4	-	30	4	0.0	1.7	-	-	10	3	7.4	7.1	-	23		
23 Aug	0.179	-	3456	2786	61	5	46	3	38	9	21	8	0.0	2.2	4.5	2.5	3.7	2.7	22	3	7.3	7.1	
24 Aug	0.180	-	3184	2878	72	6	44	4	-	24	6	0.0	1.8	-	-	23	4	7.3	7.0	-	28		
27 Aug	0.176	-	3475	2685	31	3	-	-	21	4	0.0	2.7	-	-	26	6	7.4	7.0	-	24			
28 Aug	0.159	-	3033	2149	57	2	-	-	22	4	0.0	2.3	-	-	23	3	7.2	7.0	-	20			
29 Aug	0.162	45	3133	2463	39	2	-	-	21	3	0.0	2.4	-	-	25	3	7.2	6.9	-	28			
30 Aug	0.166	-	1890	1463	89	4	37	2	-	21	4	0.0	2.8	2.5	3.7	2.7	22	3	7.2	7.0	-	26	
31 Aug	0.155	-	3106	2185	54	3	31	2	21	4	17	2	0.0	3.8	2.3	4.3	2.4	19	3	7.3	7.0	-	24
4 Sept	0.149	-	1871	1222	36	4	-	-	24	2	0.0	5.2	-	-	39	5	7.2	6.9	-	24			
5 Sept	0.155	-	2818	2104	71	4	-	-	29	4	0.0	4.1	5.4	2.1	4.6	2.4	25	3	7.2	6.9	199	261	
7 Sept	0.154	-	3024	2333	86	3	107	1	-	18	1	0.0	2.7	-	-	35	2	7.2	7.0	-	20		
10 Sept	0.153	-	2179	1609	41	6	-	-	21	4	0.0	6.4	-	-	27	6	7.4	6.8	-	-			

\* First application day after harvest.

## Applied and Runoff Water Quality from Section B of CRREL Overland Flow Test Site (cont'd).

First application day after harvest.

**Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site.**

Date	Average overland flow rate, $q$ (m <sup>3</sup> /hr/m)	Detention time (min)	Steady-state volume of wastewater, $L$ (Appl. Runoff)	BOD		TSS		TKN		NH <sub>3</sub> -N		NO <sub>2</sub> , NO <sub>3</sub> (mg/L)		Tot. P		PO <sub>4</sub> (mg/L)		Infl.		Spec. chl. (µg/m <sup>3</sup> )		T. col. (mpn/100 ml.)		E. coli (mpn/100 ml.)		T. strep. (mpn/100 ml.)			
				Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff									
23 Aug 78	0.119	45	961	822	109	-	70	4	43	22	34	15	0.0	2.1	7.8	4.8	-	43	9	7.2	603	548	36	44	-	-			
25 Aug	0.112	-	1860	1378	-	-	28	11	51	22	34	17	0.0	1.1	7.5	4.3	3.6	13	7	7.1	7.4	-	-	-	-	-	-		
29 Aug	-	-	-	-	-	-	41	10	44	23	31	17	0.0	1.3	9.0	5.3	6.6	4.7	18	6	7.3	7.5	693	578	-	-	-	-	
30 Aug	0.114	-	1302	1011	-	-	57	16	-	28	16	0.0	1.0	-	-	-	-	28	7	7.2	7.4	687	603	-	-	-	-		
1 Sept	0.117	40	1488	1219	66	17	51	13	-	36	19	0.0	2.8	-	-	-	-	29	8	7.5	7.6	-	-	-	-	-	-		
5 Sept	0.100	-	1170	723	-	-	75	20	-	30	13	0.0	1.8	-	-	-	-	35	8	7.1	7.0	589	507	-	-	-	-		
6 Sept	0.113	40	901	734	72	20	66	14	49	18	31	10	0.0	10.4	5.4	7.6	4.8	63	9	7.4	7.5	574	604	42	-	-	-		
7 Sept	0.109	-	360	273	84	9	113	5	-	9	-	0.9	-	-	-	-	-	52	4	7.3	-	507	-	-	-	-	-		
14 Sept	0.224	22	1020	818	45	15	91	9	-	31	15	0.1	0.3	-	-	-	-	34	7	7.4	7.2	556	580	-	-	-	-		
15 Sept	0.214	20	1260	908	72	20	93	15	-	31	13	0.0	1.0	8.7	5.5	6.5	4.4	50	9	7.0	6.9	547	551	-	-	-	-		
19 Sept	0.247	-	1440	1427	-	-	107	8	41	15	28	10	0.0	0.8	8.0	4.1	-	49	8	7.5	7.5	565	450	-	-	-	-		
20 Sept	0.238	-	1140	1028	42	16	112	6	-	34	14	0.0	1.1	-	-	-	-	35	7	7.1	7.1	582	458	34	29	-	-		
21 Sept	0.238	22	1260	1154	66	17	71	6	-	32	15	0.0	0.7	-	-	-	-	26	4	7.2	7.3	572	558	-	-	-	-		
25 Sept	0.215	-	840	617	-	-	64	20	39	23	31	16	0.0	1.2	10.5	7.4	9.1	6.9	27	11	7.2	7.2	597	486	-	-	-	-	
26 Sept	-	-	1080	-	-	-	25	-	-	22	-	0.1	-	-	-	-	-	23	-	7.4	-	478	-	-	-	-	-		
28 Sept	0.230	-	1020	871	51	15	52	11	-	19	7	0.2	1.2	-	-	-	-	18	7	7.1	7.0	436	347	-	-	-	-		
29 Sept	0.236	22	1080	973	62	20	79	16	-	25	12	1.2	0.8	-	-	-	-	29	8	7.1	7.2	503	522	-	-	-	-		
2 Oct	0.223	-	1200	950	-	-	127	21	-	32	14	0.0	0.9	-	-	-	-	45	10	7.4	7.4	545	440	-	-	-	-		
3 Oct	0.211	-	1260	882	-	-	141	19	41	18	34	16	0.0	0.4	9.7	4.9	4.9	4.3	42	9	7.1	7.1	554	448	-	-	-	-	
4 Oct	0.225	-	960	778	107	39	117	19	-	30	13	0.0	0.6	-	-	-	-	36	8	7.1	7.1	518	393	34	29	-	-		
11 Oct	0.232	-	3720	3218	89	26	100	20	47	26	31	19	0.0	0.7	8.3	4.8	4.8	5.7	61	13	7.3	7.1	566	503	-	-	-	-	
19 Oct	0.279	20	3380	2233	115	53	61	15	58	37	34	24	0.0	0.6	9.1	5.5	5.5	4.3	48	20	7.1	7.0	632	-	-	-	-	-	
25 Oct	0.281	-	1050	852	92	35	130	31	53	36	20	0.0	1.7	11.3	6.0	4.9	4.3	48	20	7.1	7.0	619	536	-	-	-	-		
2 Nov	0.365	19	3000	2883	86	25	97	9	39	28	18	0.0	1.7	6.7	3.3	4.5	2.4	43	8	7.3	7.4	777	769	-	-	-	-		
22 May 79	0.198	-	2916	2241	-	-	33	4	-	19	6	0.2	0.7	-	-	-	-	26	7	7.2	7.3	-	-	23	17	-	-		
23 May	-	-	3186	2349	-	-	51	-	-	25	7	0.2	1.5	-	-	-	-	28	9	7.2	7.3	-	-	-	-	-	-		
31 May	0.194	-	3463	2764	59	11	49	6	-	20	4	0.2	1.0	-	-	-	-	25	5	7.3	7.3	-	-	-	-	-	-		
1 Jun	0.215	-	2893	2183	58	15	78	7	26	7	18	4	0.1	1.3	4.4	2.0	3.4	1.8	32	7	7.3	7.3	-	-	20	22	-	-	
6 Jun*	0.203	40	3146	2466	69	40	56	14	21	13	32	19	0.1	2.0	5.0	5.2	-	-	28	4	7.3	7.3	409	479	26	-	-	-	
7 Jun	0.203	-	3630	2838	62	15	44	6	-	22	13	0.1	0.1	-	-	-	-	26	4	7.3	7.3	-	-	25	28	-	-		
8 Jun	0.203	-	1518	1187	71	11	36	6	-	24	12	0.1	3.2	-	-	-	-	64	10	7.3	7.4	-	-	-	-	-	-		
11 Jun	0.204	40	3402	2791	-	-	54	8	28	15	20	11	0.1	3.1	4.4	3.6	4.6	4.3	57	14	7.1	7.3	-	-	27	26	-	-	
12 Jun	0.215	-	3819	3279	-	-	34	3	-	15	7	0.1	1.3	-	-	-	-	28	4	7.2	7.3	-	-	-	-	-	-		
13 Jun	-	-	3132	-	77	8	53	5	-	22	12	0.1	2.0	-	-	-	-	52	5	7.4	7.5	397	317	-	-	-	-		
14 Jun	0.209	-	3245	2114	89	12	51	4	32	16	25	14	0.0	2.0	7.2	4.9	5.8	5.3	31	4	7.3	7.3	-	-	-	-	-	-	
22 Jun	-	-	3186	2657	86	8	85	5	-	21	8	0.1	1.5	-	-	-	-	37	3	7.2	7.1	-	-	-	-	-	-		
26 Jun	0.112	-	1915	1482	-	-	39	2	-	28	6	0.0	2.8	-	-	-	-	53	24	2	7.3	7.2	-	-	-	-	-	-	
27 Jun	-	-	49	4	35	2	-	-	20	3	0.0	2.9	-	-	-	-	17	2	7.2	7.0	-	-	-	-	-	-	-		
28 Jun	0.107	63	1842	1335	57	5	36	2	-	23	3	0.0	2.8	-	-	-	-	16	2	7.2	7.0	-	-	-	-	-	-		
29 Jun	0.113	1761	1334	-	28	2	-	-	3	1.2	2.2	-	-	-	-	-	-	36	3.0	12	2	7.2	7.0	375	345	26	28	-	-

**Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site (cont'd).**

Date	Average overland flow rate, $q$ ( $m^3/h/m$ )	Detention time (min.)	Steady state volume of wastewater, $L$ (ml/L)	BOD ( $mg/L$ )	TSS ( $mg/L$ )	TKN ( $mg/L$ )	$\text{NH}_3\text{N}$ ( $mg/L$ )	$\text{NO}_2\text{N}$ ( $mg/L$ )	Tot. $\text{P}$ ( $mg/L$ )	$\text{PO}_4$ ( $mg/L$ )	Turb. ( $FTU$ )	$\text{E. coli}$ ( $sp/100 \text{ mL}$ )	$\text{f. strp.}$ ( $\#/100 \text{ mL}$ )
			Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff
2 Jul 79	0.114	-	1878 1576	-	40	6	-	18	3	0.1	1.1	-	-
3 Jul 79	0.099	-	2160 1297	-	40	4	-	21	3	0.1	1.1	-	-
5 Jul 79	0.096	-	1776 975	-	36	6	-	22	2	0.1	3.4	-	-
6 Jul 79	0.099	-	2118 1258	29	6	19	3	-	23	2	0.0	3.2	-
9 Jul 79	0.099	-	1254 756	-	43	3	-	22	1	0.0	2.1	-	-
10 Jul 79	0.106	-	1896 1323	-	29	2	-	19	0	0.0	1.2	-	-
11 Jul 79	0.098	-	1812 1049	48	4	31	2	-	18	0	0.0	1.7	-
12 Jul 79	0.107	-	1800 1284	53	2	45	1	-	24	0	0.0	0.7	-
13 Jul 79	0.105	-	1932 1313	55	3	39	2	28	3	21	0.0	1.3	-
16 Jul 79	0.107	-	1514 1166	-	49	5	-	16	0	0.0	1.6	4.0	1.2x10 <sup>2</sup>
17 Jul 79	0.108	-	1734 1386	-	44	2	-	17	0	0.0	1.6	-	-
18 Jul 79	0.108	-	1728 1253	55	2	63	20	2	16	0	0.0	2.5	5.6x10 <sup>3</sup>
19 Jul 79	0.099	-	1740 1141	51	2	45	2	-	17	0	0.0	0.7	-
20 Jul 79	0.099	118	1740 1150	-	28	2	-	14	0	0.0	2.9	-	-
27 Jul 79	0.100	-	1247 839	36	19	43	8	27	7	16	5.0	9.4	2.7x10 <sup>4</sup>
30 Jul 79	0.096	-	1254 690	-	37	4	22	5	15	6.0	11.3	4.3	-
31 Jul 79	0.107	-	1806 1297	-	33	2	-	15	3	0.2	5.8	-	-
1 Aug 79	0.105	105	1806 1248	40	3	61	2	-	16	2	0.1	2.5	-
2 Aug 79	0.168	-	2619 2091	39	4	26	3	-	16	1	0.0	3.2	-
3 Aug 79	0.164	-	2700 2056	47	4	23	2	-	15	1	0.2	3.8	-
6 Aug 79	-	-	2301 2150	-	29	4	-	17	0	0.1	5.6	-	-
7 Aug 79	0.170	70	1593 1147	88	6	82	4	-	20	0	0.0	4.4	-
8 Aug 79	0.160	-	2970 2325	48	3	36	2	-	18	1	0.0	3.3	-
9 Aug 79	0.166	-	3054 2541	-	77	3	-	23	3	0.0	1.5	-	-
13 Aug 79	0.167	-	3168 2680	-	51	2	-	16	1	0.0	2.5	-	-
14 Aug 79	0.168	-	2640 2077	64	3	74	2	21	5	0.0	2.4	3.9	-
15 Aug 79	0.162	-	2116 1540	61	3	50	2	-	14	0	0.0	1.9	-
16 Aug 79	0.159	-	3240 2607	85	4	60	3	-	25	2	0.0	4.8	-
17 Aug 79	0.168	-	3168 2579	-	128	6	-	25	5	0.0	5.1	-	-
20 Aug 79	0.166	-	3033 2393	-	57	3	-	21	3	0.0	2.2	-	-
21 Aug 79	0.174	65	3294 2682	-	39	3	-	21	3	0.0	2.2	-	-
30 Aug 79	0.171	-	1890 1584	89	5	37	3	-	21	3	0.0	2.7	-
31 Aug 79	0.166	-	3177 2488	54	3	31	2	21	3	17	2.0	3.8	-
4 Sep 79	0.157	-	1914 1358	-	36	4	-	24	1	0.0	4.9	2.1	-
24 Aug 79	0.168	-	3312 72	5	44	2	-	23	4	0.0	2.3	-	-
28 Aug 79	0.167	-	2838 2289	71	7	29	3	23	1	0.0	3.1	1.7	-
29 Aug 79	0.171	-	3168 2506	86	3	107	2	-	18	1	0.0	2.3	-
30 Aug 79	0.171	-	2262 1591	41	-	-	-	21	1	0.0	5.1	-	-
11 Sept 79	0.356	-	6716 5260	-	-	-	-	24	11	0.0	2.1	19	2.7x10 <sup>4</sup>
13 Sept 79	0.385	-	1980 1744	-	-	-	-	16	16	0.1	2.6	7.3	4.3x10 <sup>4</sup>
17 Sept 79	0.102	110	1260 812	-	-	-	-	10	10	-	-	2.1x10 <sup>4</sup>	8.5x10 <sup>3</sup>
18 Sept 79	0.160	50	2651 2124	-	-	-	-	-	-	-	-	4.2x10 <sup>4</sup>	6.5x10 <sup>4</sup>

**Applied and Runoff Water Quality from Section C of CRREL Overland Flow Test Site (cont'd).**

Date	Average overland flow rate, $q$ ( $m^3/hr/m$ )	Retention time ( $min$ )	Steady state volume of wastewater, $L$ ( $m^3$ )	BOD ( $mg/L$ )		TSS ( $mg/L$ )		TKN ( $mg/L$ )		NH <sub>3</sub> -N ( $mg/L$ )		Tot. P ( $mg/L$ )		PO <sub>4</sub> ( $mg/L$ )		Turb. ( $FTU$ )		Spec. cond. ( $\mu mhos/cm$ )		Cl <sup>-</sup> ( $mg/L$ )		<i>E. coli.</i> (#/100 ml.)		<i>F. strep.</i> (#/100 ml.)				
				Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	Appl. Runoff	
19 Sept 79	0.220	42	3660	2846	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
20 Sept	0.295	30	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
21 Sept	0.382	23	6277	5435	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
25 Sept	0.373	-	3307	2709	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
28 Sept	0.382	-	7029	6054	89	39	52	20	-	40	7	33	18	25	14	0.3	0.7	5.0	3.7	-	31	13	7.1	6.9	444	369	27	26
3 Oct	0.396	-	7069	6622	87	29	46	5	-	27	17	0.0	1.4	-	-	-	-	-	-	-	35	18	7.3	7.0	-	-	-	-
5 Oct	0.384	-	7544	6588	61	17	32	7	-	22	14	0.0	0.4	-	-	-	-	-	-	-	39	18	7.2	7.3	-	-	-	-
10 Oct	0.386	-	5564	4906	77	31	36	10	35	21	28	15	23	14	0.0	0.8	4.8	3.1	24	8	7.2	7.1	388	327	23	21		
12 Oct	0.386	-	6574	5823	102	37	39	9	-	30	23	0.0	1.7	0	1.7	5.7	4.6	4.6	3.7	28	14	7.4	7.1	491	412	32	28	
17 Oct*	0.381	-	6851	5876	-	84	12	49	32	35	28	0.2	1.1	8.0	6.1	5.3	5.3	38	19	7.7	7.4	536	424	31	31	-	-	
19 Oct	0.381	-	6712	5749	143	60	79	15	-	35	30	0.2	0.0	-	-	-	-	44	20	7.3	7.3	-	-	-	-	-	-	
23 Oct	0.304	-	4928	4140	-	65	6	39	20	28	15	0.0	4.3	6.2	4.7	5.2	4.5	35	11	7.2	7.2	452	409	28	28	2.4x10 <sup>6</sup>	6.1x10 <sup>5</sup>	
24 Oct	0.238	-	744	681	-	161	36	-	29	10	0.0	3.5	-	-	-	-	92	36	7.0	7.4	-	-	-	-	-	-		
29 Oct	0.113	-	1572	1281	-	88	10	46	16	33	12	0.0	8.3	8.4	4.3	5.6	3.3	33	6	7.2	7.1	555	449	35	35	2.5x10 <sup>6</sup>	5.6x10 <sup>5</sup>	
31 Oct	0.110	-	1758	1358	114	21	94	7	-	33	12	0.0	8.3	-	-	-	42	9	7.4	7.3	-	-	-	-	31	31		
2 Nov	0.108	-	1204	1025	101	12	67	8	-	30	11	0.1	7.9	-	-	-	37	7	7.2	7.1	-	-	-	-	31	31		

\* First application day after harvest.

## APPENDIX B: ANALYTICAL TECHNIQUES

Suspended solids analyses (total and volatile) were performed according to Millipore Bulletin AB312, (1975) in which the glass fiber filter technique (drying at 103°-105°C for total solids and igniting at 550°C for volatile solids) was used. This is basically the same procedure as in *Standard Methods* (APHA 1975, p. 97), except that it is more specific. One modification of this procedure was to momentarily lift the filter from the holder after the sample had filtered through and then to replace it for approximately one minute. This was necessary to release a vacuum causing a film of water to remain directly under the filter. It was found that if not dried sufficiently, the filter would stick to the aluminum pan when dried in the oven. This modification, which proved to be very effective, was used in both filter preparation and sample filtration. The only other modification was that the filters were weighed to the nearest 0.01 mg instead of 0.1 mg as stated in the Millipore procedure.

Biochemical oxygen demand (BOD) measurements were obtained using the Winkler method, "Oxygen Demand (Biochemical)" in *Standard Methods* (1975, p. 543-549). Dissolved oxygen measurements (initial and five day) were made according to "Azide Modification" of the Iodometric Method, *Standard Methods* (1975, p. 443-445). Most reagents used were purchased (lot numbers and new bottles recorded) to ensure consistency. The phosphate buffer was prepared as stated in *Standard Methods* (1975, p. 545) in 100-mL amounts, refrigerated when not used and replaced approximately every month. Dilution technique 2 was used (p. 547). Reagents were added gently down the necks of the BOD bottles using repeater pipettes. Glucose-glutamic acid, dilution water and seeded blanks were run with each test.

Turbidity measurements were obtained using a Hach turbidimeter, model 2100A, according to the EPA approved procedure described in "Wastewater Analysis Handbook" (Hach Chemical Co. 1978, p. 592). Results were expressed in Jackson Turbidity Units (JTU). Samples with a turbidity reading greater than 40 JTU were *not* diluted. The turbidimeter was recalibrated before each sample, using the prepared latex standard.

Fecal coliforms and fecal streptococcus bacteria were enumerated according to procedures described in *Standard Methods* (1975, p. 937-939 and 944-945, respectively) and Millipore's "Biological Analysis of Water and Wastewater," Application Manual AM 302, p. 34-35. Millipore's 2-mL ampoules of M-FC broth were used for fecal coliform test and agar plates of

*M-Enterococcus* were prepared for the fecal streptococcus test. One-hundred-milliliter volumes of varying sample dilutions (at least 3 different dilutions/sample) were filtered for each test. Millipore's incubator, which has the ability to maintain a critical narrow temperature range (44.5°C ± 0.2°C), was used.

The pH of the samples was taken using a Markson 1808 probe and an Orion 801 Ionalyzer. The probe was calibrated daily with pH 7.00 and 4.00 buffers. Values for pH were read after the probe was left in solution 1 minute to ensure consistency.

*Nitrate-nitrogen* was analyzed on the Technicon Auto Analyzer II (AAII) using Technicon's "Automated Cadmium Reduction Method." Two procedures were employed depending on amount of NO<sub>3</sub>-N present. Technicon Industrial Method (T.I.M.) 246-731 ("3 in. Dialyzer Method") was used for samples ranging from 0 to 50 mg L<sup>-1</sup> NO<sub>3</sub>-N, and T.I.M. 271-73W ("24-in. Dialyzer Method") for samples between 0 to 1 mg L<sup>-1</sup> NO<sub>3</sub>-N.

Samples were tested for *ammonia-nitrogen* (NH<sub>3</sub>-N) using the Technicon AAII. The "Salicylate/nitroprusside Method" was used according to T.I.M. procedure 329-74 W/A (revised Jan 1976) for NH<sub>4</sub>-N ranges from 0 to 50 mg L<sup>-1</sup>

The Technicon AAII "Block Digestion Method" was used for analysis of *Kjeldahl N* within the range of 0 to 50 mg L<sup>-1</sup>. The digestion procedure used was T.I.M. 376-75 W/B (revised March 1977) and analysis procedure T.I.M. 329-74 W/B (also revised March 1977).

*Chloride* was analyzed on the Technicon AAII using the "Thiocyanate Method" for ranges of 0 to 35 mg L<sup>-1</sup>, according to T.I.M. 99-70 W (1973).

*Total P* was analyzed on the Technicon AAII using the "Block Digestion Method" (molybdenum blue analysis for ranges of 0 to 10 mg L<sup>-1</sup>). Digestion was carried out according to T.I.M. 376-75 W/B (1977) and analysis followed T.I.M. 329-74 W/B procedure (revised March 1977).

*Ortho-P* was analyzed using a Coleman Junion Spectrophotometer and the "Manual Molybdenum Blue Method" for ranges of 0 to 0.11 mg L<sup>-1</sup>, according to Hach Chemical Company's "Water and Wastewater Analysis Procedure."

*Specific Conductance* was obtained using a resistivity bridge for ranges between 100 to 1000  $\mu$ hos cm<sup>-1</sup>.

## APPENDIX C: DETERMINATION OF SCOUR VELOCITY

An estimate of the scour velocity for overland flow systems was obtained from sedimentation theory. According to Metcalf and Eddy (1972), the horizontal velocity that will just produce scour in a settling basin can be determined from the following relationship:

$$V_s = \left[ \frac{8k(s-1)gd}{f} \right]^{1/2} \quad (C1)$$

where  $V_s$  = horizontal scour velocity ( $\text{m s}^{-1}$ )  
 $s$  = specific gravity of particles  
 $d$  = diameter of particles (m)  
 $k$  = constant which depends on the type of material being scoured  $\approx 0.06$   
 $g$  = gravitational constant =  $9.81 \text{ m s}^{-2}$   
 $f$  = Darcy-Weisbach friction factor = 0.025.

Assuming a particle size of 0.1 mm and a specific gravity of 1.1 the scour velocity was calculated to be  $0.0434 \text{ m s}^{-1}$  or  $2.6 \text{ m min}^{-1}$ . This appears to be a reasonable estimate based on the performance of the CRREL overland flow site. Suspended solids removal began to decrease slightly at a detention time of 20 minutes, which is equivalent to a velocity of  $1.5 \text{ m min}^{-1}$ .

The limiting overland flow rate that will produce a scour velocity ( $V_s$ ) of  $2.6 \text{ m min}^{-1}$  can be determined from eq 13 as follows:

$$\bar{T} = \frac{0.078 L}{1/3} = \frac{L}{V_s}.$$

Solving for  $q$ ,

$$q = \frac{0.078 V_s}{S^{1/3}} = \frac{0.078(2.6)}{S^{1/3}} = \frac{0.20}{S^{1/3}}.$$

Therefore, in order to avoid resuspension of solids and scour, the average overland flow rate should be limited to  $0.20/S^{1/3}$  or less.

A facsimile catalog card in Library of Congress MARC format is reproduced below.

**Martel, C.J.**

Development of a rational design procedure for overland flow systems / by C.J. Martel, T.F. Jenkins, C.J. Diener and P.L. Butler. Hanover, N.H.: U.S. Cold Regions Research and Engineering Laboratory, Springfield, Va.: available from National Technical Information Service, 1982.

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